Impact response of double-layer Steel-RULCC-Steel sandwich panels: experimental, numerical and analytical approaches

Wei ZHANG[1], Zhenyu HUANG*[2], Ren LI[3], Xiaolong ZHAO[4], Jianqiao YE[5]

1 *Guangdong Provincial Key Laboratory of Durability for Marine Civil Engineering, Shenzhen University. L1-1408, Shenzhen, China 518060. Email: zhangwdnv@gmail.com*

2 *Guangdong Provincial Key Laboratory of Durability for Marine Civil Engineering, Shenzhen University. A412, College of Civil and Transportation Engineering, Shenzhen, China 518060. Email: huangzhenyu@szu.edu.cn*

3 *Guangdong Provincial Key Laboratory of Durability for Marine Civil Engineering, Shenzhen University. L1-1508, Shenzhen, China 518060. Email: fishermanlr@163.com*

4 *Guangdong Provincial Key Laboratory of Durability for Marine Civil Engineering, Shenzhen University. L1-1508, Shenzhen, China 518060. Email: 347479165@qq.com*

5 *Department of Engineering, Lancaster University. Lancaster LA1 4YR, UK. Email: j.ye2@lancaster.ac.uk*

Abstract

The present study conducts experimental, numerical and analytical investigations on the responses of double-layer Steel-RULCC-Steel sandwich panels subjected to concentrated impact loading. Seven full-scale SCS panels are designed and fabricated with different number of concrete layers, degree of composite action, type of shear connectors, and proportion of added rubber powder. The influences of these design parameters on the failure mode and response behavior are quantified and discussed. Advanced FE simulation is performed in LS-DYNA to extract more information on the strains, stresses, and energy absorption of the panel during the impact. Finally, a single-degree-of-freedom (SDOF) model and a two-degree-of-freedom (TDOF) model are developed to predict displacement-time and load-time responses of the double-layer SCS panels based on the quasi-static load-displacement relationship proposed also in this paper. The comparisons with test results demonstrate that the SDOF model overpredicts the peak deformation of the panel if the hammer weight is much larger than the effective panel weight. In contrast, both the FE model and TDOF model provide a much more accurate prediction on the impact responses of double-layer SCS panels, including the peak impact force, peak deformation, and residual deformation.

Keywords: *Impact, Steel-Concrete-Steel, RULCC, Double-layer, SDOF, TDOF*

1. Introduction

During the design service life, an engineering structure may have to carry not only static, but also impact loads such as impacts from moving vehicles, aircrafts or ships. Traditional reinforced concrete (RC) structures are generally vulnerable to crush or crack when they are under an impact load, as the steel rebars are much less effective in restraining the concrete that is subjected to a complex stress state (Adhikary et al. 2012; Zhan et al. 2015; Lee et al. 2021). The caused damage is hard to be quickly repaired and may pose huge security risks to the society. In contrast, Steel-Concrete-Steel (SCS) sandwich composite structures exhibit both excellent tensile and excellent compressive performances (Remennikov et al. 2013; Wang et al. 2015). Under impact loads, the steel plates can effectively prevent penetration of the impactors, while the concrete core work as an energy dissipation layer, leading to excellent impact resistance (Zhao and Han 2006; Wang et al. 2016; Sohel et al. 2003).

Various studies have been conducted to investigate the dynamic behavior of SCS structures subjected to impact loadings. Jung et al. (2019) proposed a criterion to determine critical velocity of the impact, which was numerically verified by considering strain rate effect and material damage in the FE model. Lu et al. (2021) proposed a flat steel-concrete-corrugated steel sandwich panel and experimentally studied its dynamic response under impact loading. The impact energy was found mainly dissipated by the concrete core, then by the corrugated plate and the flat plate. Wang et al. (2021b) improved the impact resistance of SCS beams by welding additional stiffeners on the tension plate. All the specimens in the experiment showed a flexural failure with a plastic hinge generated at the mid-span. Analytical spring-mass models have been proposed for impact analysis of RC structures (Fujikake et al. 2009; Sha and Hao 2014; Pham and Hao 2016, 2018). Similar methods were also applied to SCS structures. Guo and Zhao (2019) presented a single-degree-of-freedom (SDOF) model by ignoring the contact process to predict impact response of SCS panels. As an improvement, Wang et al. (2021a) established a two-degree-of-freedom (TDOF) model that can predict the impact force and local deformation of the panel. For both the SDOF and TDOF models, quasi-static resistances and stiffnesses of the structure are required to be determined first. All the studies discussed above adopt the ordinary concrete as the core material for the sandwich structures applied as submerged tube tunnels (Narayanan et al. 1997), shear walls, and protective structures, etc. The sandwich structures with ultra-high performance concrete (UHPC) can be adopted for high performance composite structures, such as nuclear shielding walls. Lin et al. (2020) studied the failure mechanism and failure patterns of SCS sandwich beams with steel fiber-reinforced UHPC. Compared to sandwich beams with ordinary concrete, sandwich beams with UHPC tend to fail by flexural failure rather than shear failure. UHPC as core material decreases the slippage between concrete and steel plates, and the steel fibers are effective in preventing the development of cracks. The impact studies of sandwich structures with UHPC are not found in existing references. In addition, SCS structures are potentially designed for marine and offshore structures, such as ship hulls, bridge decks, liquid containment, and offshore platforms, etc., the application of which requires the weight of the concrete core to be light (Bergan and Bakken 2005). To solve this problem, lightweight concrete of density less than 1500kg/m^3 serves as a good choice. Liew and Sohel (2009) designed a lightweight SCS sandwich system using J-hooks as shear connectors. The designed SCS beams and panels were experimentally (Liew et al. 2009; Sohel and Liew 2014) and numerically (Sohel et al. 2015) studied to evaluate their impact performance under drop weight impact. The results showed that the lightweight concrete exhibits brittle behavior and may crack into many pieces at the impact event, and using 1% to 2% volume fraction of fiber in concrete core could reduce the cracks significantly and enhance the overall integrity of the sandwich structure.

The composite action between steel plates and concrete core of SCS structures is guaranteed by the mechanical shear connectors welded on steel plates. Various types of shear connectors have been proposed in existing studies, such as overlapped headed studs (Oduyemi and Wright 1989), Bi-steel (Foundoukos 2005), angle shear connectors (Guo et al. 2020), interlocked J-hook connectors (Liew and Sohel 2009) and bolt connectors (Yan et al. 2020), etc. The J-hook connectors are effective in preventing tensile separation between steel face plates due to the interlock effect. However, SCS composite structures with pure J-hook connectors may lead to a congested reinforcement, making concrete casting and assembly of curved SCS structure infeasible. As an improvement, Zhang et al. (2020) proposed a new design of SCS structure with hybrid connectors consisting of J-hooks and overlapped headed studs. In addition, the beams and panels investigated above are all made of a single layer concrete and two steel face plates, by which the materials may not be effectively and efficiently utilized to resist an impact. To make efficient use of the materials through composite actions, Huang et al. (2021b) developed a novel double-layer SCS sandwich panel using lightweight high ductility cement composite and multi-layer structural optimization. The test results proved that the double-layer SCS panel had higher ductility and impact resistance than the single-layer SCS panel. The authors also proposed to add rubber powder into the ultra-lightweight cement composite (ULCC) to improve the impact resistance of SCS panels, since rubber power has excellent energy absorption capacity (Xue and Shinozuka 2013; Liu et al. 2012). However, the addition of rubber powder may reduce the compressive strength and workability of concrete (Huang et al. 2020, 2021a). To overcome this issue, silica fume and steel or polymer polyethylene (PE) fibers were added in the mix (Gupta et al. 2015; Ali et al. 2017; Guo et al. 2018).

The present study fabricates seven full-scale Steel-rubberized ULCC (RULCC)-Steel sandwich panels with varying material and geometric parameters, and tests them under drop hammer impact. To extract more information on the strain, stress and energy absorption of the panel, the study conducts advanced FE simulation in LS-DYNA. Finally, the authors develop both analytical SDOF and TDOF models to predict the deformation and impact force for double-layer SCS panels. The test results are adopted to validate the FE, SDOF and TDOF models.

2. Experiment

This study conducts a full-scale experimental program to examine the dynamic behavior of double-layer SCS panels under impact loading. Figure 1a and b show the configuration of single-/double-layer SCS panels, respectively.

2.1 Test specimens

In total, seven full-scale SCS panels are designed and fabricated with different number of concrete layers, degree of composite action, type of shear connectors, and proportion of added rubber powder. Table 1 lists the geometric parameters, the selections of which are based on the structure in practical application. The name of each specimen consists of three parts. The first part, SULCS, SR5ULCS or SR10ULCS, indicates, respectively, that the material of the sandwich core is ULCC, ULCC with 5% rubber powder in volume or ULCC with 10% rubber powder in volume. The second part, 100, 150 or 200, indicates that the spacing between the shear connectors. The third part, 9(SH), 6(DH) or 6(DJ), indicates that the thickness of the steel plate is either 9mm or 6mm, where S and D are, respectively, for Single and Double Layer and H and J represent Hybrid or pure J-hooks. Both the single-layer and double-layer panels have the same total thickness of steel and total thickness of concrete, ie., they have the same steel and concrete volume fraction. The spacing of the shear connectors reflect the degree of composite action, η, which is calculated as the ratio of the overall shear strength of the connectors within a shear span to the average tensile strength of the steel plate.

$$\eta = \frac{n_s V_s}{f_{yp} t_p \, L_s/2} \tag{1}$$

where n_s is the number of shear connectors within the shear span; V_s is the shear strength of a single shear connector (Huang et al. 2021b); f_{yp} is the yield strength steel plates.

2.2 Material properties

The study designs three mix proportions of ULCC, namely the ULCC without rubber, the ULCC with 5% and 10% volume proportion of rubber powder (R5ULCC and R10ULCC). The rubber powder replaces the same volume of fine aggregates (fly ash cenospheres) in the mix. Table 2 lists the mix proportion of the ULCC and RULCC. Figure 2 displays the appearance of each material component. In order to improve the ductility of the concrete, each mix is added with 5.8kg/m^3 (0.6% volume proportion) PE fiber. High-water reducing agent is added to ensure the fluidity of the concrete that is 220-230mm, measured according to BS EN 1015-3 (1999), as shown in Figure 2h. Table 3 lists the material properties of ULCC, R5ULCC and R10ULCC. The compressive cube strength is tested according to the Chinese standard GB/T50081 (2019), and the compressive cylinder strength is tested according to ASTM C39/39M (2021), and the tensile strength is tested according to JSCE (2008). The compressive strengths and elastic modulus decrease with the increase of the volume proportion of rubber powder. Thus, the addition of rubber powder decreases the material strength, but increases its flexibility. Table 4 lists the material properties of the steel components tested according to ASTM E8/E8M (2011).

2.3 Test set-up and measurement

The impact test is conducted on the STLH-50000 drop hammer impact test machine, with a maximum counterweight of 1012kg. Figures 3a and 3b show the test set-up of the drop weight impact machine. The hammer is released 5m above the panel, generating a maximum impact energy of 50000J. The head of the drop hammer is round shaped with a diameter of 100mm. Figure 3c displays the clamping device for the frame-rigid supporting floor and for the specimen-support frame. The SCS panel is simply supported on the four circular steel bars welded to the foundation. All the seven SCS panels are subjected to the same impact energy, 50000J. The experiment measures the impact force, deformation of the panel, strains of the steel plate, and records the whole impact process. Figure 4 shows the measurement scheme.

2.4 Failure mode

The entire impact process can be divided into four main stages based on the status of the hammer.

Stage I: Free fall of the hammer. At the time before the hammer contacts with the surface of the panel, the velocity of the hammer reaches the maximum.

Stage II: In contact with the panel. The hammer starts to contact with the panel and transfer the impact force and energy to the panel, leading to global bending and local indentation of the panel. The velocity of the hammer reduces to zero when the deformation of the panel reaches the maximum.

Stage III: Rebound of the hammer. After the velocity of the hammer reduces to zero, it keeps in touch with the panel until the elastic deformation of the panel has been fully recovered. The hammer continues to rebound and starts to separate from the panel. The hammer stops to rebound when the rebound velocity reduces to zero.

Stage IV: Follow-on free fall and rebound of the hammer (repeated stage I-III). Since the drop height is significantly reduced compared with that of the first impact, the secondary impact energy is very small and can be neglected.

Figure 5 shows the two typical failure modes observed at the end of the test: local indentation with and without fracture. For the single-layer panel, SULCS-150-9(SH), the impact area is severely indented. The top steel plate is partly fractured by the drop hammer and the length of the crack is around 20% of the perimeter along the indentation. For the double-layer panel, SULCS-150-6(DH), although local indentation is observed, the top steel plate is not fractured. This indicates that the double-layer SCS panel exhibits better protective performance to impact loading than the single-layer panel. Compared to the single-layer panel, the middle steel plate of the double-layer panel also contributes to resisting the impact load. The middle steel plate distributes the load to a larger area in the second layer of the panel, thus more materials are motivated to absorb the impact energy. The same failure modes are observed from all the double-layer SCS panels in this study. The experiment has adopted waterjet to cut the panel, SR10ULCS-150-6(DH), after the impact test, which is shown in Figure 5c. The panel clearly exhibits both global flexural deformation and local indentation. The top steel plate under the drop hammer is severely yielded.

2.5 Strain distribution

The experiment measures the strains on the surfaces of the top and bottom steel plates whose yield strain is about $2000\mu\varepsilon$. The measured strain distribution exhibits similar characteristics for each SCS panel. Figure 6 shows the strain-time curves of SULCS-150-6(D). The positions of the strain gauges are shown in Figure 4. On the top steel plate, both the peak and residual strains measured by SS1 and SS3 are high of $20000\mu\varepsilon$ or above, showing that the materials at those locations have experienced notable plastic deformation. The peak and residual strains measured by SS5 are around

4200με and 2500με, respectively, indicating the material at this point is yielded but not as serious as those at SS1 and SS3. Due to the Poisson's effect, the strain measured by SS6 in the tangential direction is negative (i.e., compressive). Considering the positions of SS5 and SS6, it is concluded that the steel material within a radius of 141mm from the impact center has yielded, while the material outside this region remains elastic. The bottom steel plate exhibits bulging deformation. The maximum peak strain and the residual strain measured at the bottom steel plate are around 6500με and 5000με, respectively, which are much smaller than the maximum values at the top steel plate. Thus, the yielding of the bottom steel plate is not as serious as that of the top steel plate. The reason is that when the load transfers from the top steel plate to the middle steel plate and then to the bottom steel plate, a larger volume of the materials have participated in absorb the energy of the impact, leading to lower stress and strain in the bottom steel plate.

2.6 Influence of number of layers

Table 5 lists the impact parameters and test results of the seven specimens. Figure 7 compares the load-time responses and displacement-time responses for the four groups of specimens with different layer numbers, degrees of composite action, types of shear connectors and volume proportions of rubber powder. As shown in Figure 7a, the load-time and displacement-time responses of SULCS-150-6(DH) and SULCS-150-9(SH) are very close to each other, with almost the same peak load and peak displacement. However, the single-layer panel is damaged more severely than the double-layer panel, as the top steel plate is partly fractured with the length of the crack around 20% of the perimeter along the indentation. In practice, the fracture may present as safety hazards, which are expensive to repair. The double-layer SCS panel has higher impact resistance and is more cost-effective to be used as a protective structure than the single-layer SCS panel does.

2.7 Influence of degree of composite action

The degree of composite action, η, is reflected by the spacing between the shear connectors. Figure 7b compares the load-time and displacement-time responses of SULCS-100-6(DH), SULCS-150-6(DH) and SULCS-200-6(DH), whose

η are, respectively, 1.00, 0.64 and 0.37. The peak impact forces of the above three specimens are 1063.4kN, 960.2kN, and 907.7kN, respectively, and their respective peak displacements are 32.8mm, 43.3mm, and 47.4mm. These results demonstrate that the impact force increases while the displacement decreases with the increase of η. This is because the SCS panel with smaller degree of composite action is more flexible and ductile, due to the bond-slip effect between the steel plate and the concrete (Huang et al. 2021b). Therefore, the panel with weaker composite action shows higher deformation capability, thus sustains smaller impact force than the panel with stronger composite action when they are subjected to the same impact energy.

2.8 Influence of type of shear connectors

The SCS panel with only J-hooks as shear connectors exhibits excellent impact resistance due to the “interlock” effect provided by the J-hook pairs. Figure 7c compares the load-time and displacement-time responses of SULCS-150-6(DH) with hybrid shear connectors and SULCS-150-6(DJ) with J-hooks. The peak impact forces of these two panels are 960.2kN and 874.3kN, and the peak displacements are 43.3mm and 45.5mm, respectively. SULCS-150-6(DH) suffers larger impact force but displays slightly smaller deformation. Thus, both the panel with hybrid shear connectors and the panel with J-hooks show almost the same energy absorption ability under impact loading. In addition, the panel with hybrid shear connectors has larger transverse shear resistance than that with J-hooks (Zhang et al. 2020) and is simple to fabricate. As a result, the panel with hybrid shear connectors provides a good alternative in practical engineering.

2.9 Influence of rubber powder

Rubber powder is added into the ULCC to evaluate its influence on the impact resistance and energy absorption of the SCS panels. Figure 7d compares the load-time and displacement-time curves of the panels with different volume proportions of rubber powder. Compared to SULCS-150-6(DH), which does not contain rubber powder, SR5ULCS-150-6(DH) with 5% volume proportion of rubber powder suffers smaller impact force but has a larger deformation. This is due to the fact that the addition of rubber powder reduces elastic modulus of the concrete, leading to a higher energy absorption ability of the composite panel. However, SR10ULCS-150-6(DH) with 10% volume proportion of rubber powder has irregular results. The impact force and displacement responses remain similar to those of the specimen without rubber addition. Thus, the curves in Figure 7d do not show the expected trend probably because of the test deviations caused by unexpected factors, such as the mixing of concrete (the aggregate, rubber powder, fiber, etc.) is uneven, the cracks developed during the impact are random and discrete, the loading and measuring may introduce some deviations, etc. However, both the numerical simulation and proposed analytical model have excluded the influences of these factors. The volume proportion of rubber powder is the only factor affecting the impact responses of these three panels. The details on the numerical results and analytical results are discussed in Section 4.5.

3. Numerical Modelling

The study conducts numerical simulation to further investigate impact resistance of the SCS panels and extract more information on the stress and strain distribution, as well as the damage of the concrete during the impact process. The geometric model of the panel is modelled in SOLIDWORKS, and imported into HYPERMESH for refined meshing. The model is then imported into LS-DYNA for simulation and post-processing.

3.1 Material model of concrete

The CSCM model, *mat_159, is used to define the material properties of ULCC and RULCC. The CSCM model is a smooth and continuous surface cap model that is available for solid elements in LS-DYNA. With the consideration of material hardening, damage and strain rate effects, the CSCM model is widely applied in the field of simulating steel-concrete composite structures and reinforced concrete subjected to low-velocity impact. The damage of concrete is initiated when the energy-type terms, τ_c and τ_t, have exceeded the damage thresholds, τ_{c0} and τ_{t0}, for compressive and tensile stress, respectively. τ_c and τ_t are defined based on Eq. (2) and Eq. (3).

$$\tau_c = \sqrt{0.5\sigma_{ij}\varepsilon_{ij}} \tag{2}$$

$$\tau_t = \sqrt{E_c\varepsilon_{max}^2} \tag{3}$$

where σ_{ij} and ε_{ij} are the stress and strain tensor, respectively; E_c is the elastic modulus of concrete; and ε_{max} is the maximum principal strain. Since the initial damage threshold is coincident with the shear plasticity surface, there is no need to specify the values of τ_{c0} and τ_{t0}.

After the concrete is damaged, the CSCM model converts the visco-plastic stress tensor without damage, σ_{ij}^{vp}, to the stress tensor with damage, σ_{ij}^{d}, through the following equation.

$$\sigma_{ij}^{d} = (1-d)\sigma_{ij}^{vp} \tag{4}$$

where d is a scalar damage parameter ranging from 0 to 1, i.e., from no damage to complete damage of the concrete.

The CSCM model requires to specify 37 variables to determine the yield surface, hardening cap, damage rule, and rate effects. The variables related to the yield surface and hardening cap are calculated automatically with the input of the basic material properties of the ULCC and RULCC in this study, such as the compressive strength, tensile strength, elastic modulus, shear modulus, density, etc. These material properties are obtained through standard material tests, as listed in Table 3. The other variables related to the damage rule and rate effects are taken from existing literatures (Meng 2012).

3.2 Material model of steel

The material model of steel plates and shear connectors adopt the piecewise linear plasticity material model *mat_024. Table 4 lists the yield strength, yield strain, ultimate strength and ultimate strain of the steel materials obtained from coupon tests. The strain-rate effect of steel material takes the Cowper-Symonds model in Eq.(5).

$$\frac{\sigma_d}{\sigma} = 1+\left(\frac{\dot{\varepsilon}}{D}\right)^{1/q} \tag{5}$$

where σ_d is the dynamic stress at a uniaxial strain rate $\dot{\varepsilon}$. The coefficients D and q are set as 40.4 and 5 respectively (Zhao et al. 2018). As the drop hammer and support rollers are within elastic deformation throughout the impact loading process, the material properties of drop hammer and support rollers are simplified as elastic, and the rigid material model *mat_020 is adopted with an elastic modulus of 210GPa and a Poisson's ratio of 0.3.

3.3 Modelling of shear connectors

The J-hooks and headed studs in the FE model require special treatment, as detailed modelling of the geometry of the shear connectors is time-consuming and would likely lead to convergence problems during the iteration. Thus, a simplification is proposed to define the coupling relationship of the shear connectors by the nonlinear spring element, *SPRING-NONLINEAR-ELASTIC. Both the J-hooks and the overlapped headed studs are represented by two steel bars connected with the nonlinear spring element, as shown in Figure 8. The load-displacement curves of the spring element are obtained by testing the SCS unit with J-hooks and overlapped headed studs, respectively.

3.4 Mesh, contact and boundary condition

The element type of the drop hammer, concrete, and support rollers are meshed by the default 8-node solid element which uses one-point integration plus viscous hourglass control. The element type of the steel plates is the Belytschko-Tsay shell element. Figure 8 shows the FE model, where a quarter model is built due to symmetry. From the mesh sensitivity study, the element size within the impact region (260mm*260mm) is determined as 7mm*7mm, while a coarser element of 14mm*14mm is used outside this region. The element number in the thickness direction of each concrete layer is 8. The interaction between two different components in this study is defined by the automatic surface-to-surface contact, which requires to specify the static friction coefficient (c_{fs}) and the dynamic friction coefficient (c_{fd}). For the interactions between steel plate and concrete, between shear connector and concrete, c_{fs} and c_{fd} are set as 0.7 and 0.5, respectively. For the interactions between hammer and steel plate, between rollers and steel plate, the respective c_{fs} and c_{fd} are 0.5 and 0.2.

3.5 Validation of the FE model

Figure 9 compares the residual deformation of the top and the bottom steel plate between the test and the FE result for each of the double-layer SCS panels. The comparisons show that the FE results have a very good match with the test results for the residual deformation of the bottom steel plate; while for indentation of the top steel plate, the FE results are slightly larger than the test results. This is due to that the material of the top steel plate around the impact region is severely yielded, and the strain rate effect also probably affects the accuracy of the simulation. However, this difference is small and negligible. In addition, the study compares the load-time response and the vertical displacement at the center of the bottom surface of the panel with the test results, the details of which will be discussed in Section 4.5. It can be concluded from these comparisons that the proposed FE model is sufficiently accurate to be used to simulate the impact tests of the panels.

3.6 Development of cracks

Figure 10 shows the development of crack for SULCS-150-6(DH) during the impact. The fringe level shown by the diagrams indicates the status of crack, where “1” indicates the material has cracked and “0” indicates the material is intact. At the initial contact between the drop hammer and panel, the concrete underneath the hammer starts to be compressed with the appearance of minor damage. Then, a punching cone is gradually developed which serves as the main part to transfer the impact loading. The damage status of the concrete within the punching cone becomes severe. As the hammer drops down, the load is distributed to a larger area inside the panel and more concrete is severely damaged when the maximum deformation is reached. Afterwards, the hammer starts to rebound, and part of the deformation of the panel is restored, but the damage status of the cracked concrete keeps unchanged.

3.7 Energy consumption

Figure 11 compares the energy consumption ratio of each component after impact for the four groups of specimens with different layer numbers, degrees of composite action (η), types of shear connectors and volume proportions of rubber powder. The following information can be obtained from the figure: (1) The steel plate and the concrete consume more than 95% of the impact energy. The energy is consumed in the form of yielding of the steel plate and cracking of the concrete. (2) The energy consumption ratio of concrete for the double-layer panel (52.4%) is slightly higher than that for the single-layer panel (51.4%), due to the reason that the middle steel plate spreads the load to a larger area inside the panel and more concrete is involved in absorbing the energy. (3) A decrease of η increases the energy consumption ratio of steel plate but decreases the energy consumption ratio of concrete. The reason is that a fully composite SCS panel tends to experience transverse shearing, generating a widely opened critical diagonal crack and many small diagonal cracks in the concrete; while a partially composite SCS panel tends to develop a few flexural cracks in the concrete (Huang et al. 2021b; Zhang et al. 2020). Thus, the concrete consumes larger portion of energy in the SCS panel with larger η; and thereby, the portion of energy consumed by the steel plate reduces with the increase of η. (4) The J-hooks consume a slightly larger portion of energy than the hybrid shear connectors (4.6% vs 2.0%), demonstrating that J-hooks are more effective in resisting impact loads due to the "interlock" effect. (5) The addition of rubber in concrete increases the energy consumption ratio of the concrete, leading to the reduction in the energy consumption ratio of steel plate, due to the hyperplastic and excellent energy absorption capacity of the rubber.

To better reveal the effectiveness of rubber powder, the authors have conducted deeper research in another paper on the impact resistance of double-layer Steel-RULHDCC sandwich panels subjected to repeated impact loads (Huang and Zhang 2020). In this paper, the waterjet is adopted to cut the panel after test and view the cross section of the panel. The comparison shows that there are fewer concrete cracks in the cross-section with higher volume proportion of rubber. Both the ULCC and rubber powder are effective to absorb the impact energy. The increase of rubber powder would decrease the energy transferred to the ULCC, then the ULCC would keep better integrity and be capable to take more impact loads. In summary, the shear connectors and rubber powder are effective to improve the impact resistance of the structure from the mechanical side and material side, respectively. As the rubber powder is made by grinding the rubber waste produced by scrap tires, the cost is much cheaper (Huang et al. 2021c). How to optimize the content of rubber and shear connectors is of great interest, and a future study is needed to address this issue.

4. Analytical study

The impact response of a double-layer SCS panels can be predicted by an equivalent mass-spring-damper model. Depending on whether the drop hammer is modelled independently, the analytical model can be classified as a single-degree-of-freedom (SDOF) model or a two-degree-of-freedom (TDOF) model. The prerequisite of both two models is to obtain the quasi-static resistances and stiffnesses of the double-layer SCS panel.

4.1 Resistances under quasi-static loading

Figure 12a plots the load-deformation profile of the double-layer SCS panel subjected to concentrated punching load. Based on the observations from tests, a punching cone with an inclination angle of 60° is formed. Figure 12b shows the 3D punching cone extracted from Fig. 12a and the mechanism of load transfer. The external punching load is resisted by the steel plates V_p, the concrete V_c, and the shear connectors V_s. The total deformation at the bottom-center of the panel consists of the global bending deformation and the local bulging deformation, as shown in Figure 12c. Thus, the stiffness of the panel can be predicted by using a tandem spring model with a global stiffness k_g and a local stiffness k_b. Figure 12d plots the idealized load-displacement curve of the double-layer SCS panel under quasi-static loading based on previous studies (Huang et al. 2021b; Zhang et al. 2021). The first peak resistance at point A indicates the punching shear failure of the concrete core, the second peak resistance at point C indicates the punching shear fracture of the top steel plate, and the third peak resistance at point E indicates the punching shear fracture of the middle steel plate.

At point A, the punching cone is initially formed and the first peak resistance P_1 consists of the contributions of the top steel plate V_{pt}, the upper layer concrete V_{c1}, the lower layer concrete V_{c2}, the upper layer shear connectors V_{s1}, and the lower layer shear connectors V_{s2}. i.e.,

$$P_1 = V_c + V_s + V_p = V_{c1} + V_{c2} + V_{s1} + V_{s2} + V_{pt} = V_{c1} + V_{c2} + V_{s1} + V_{s2} + 0.33\sqrt{f_c}\frac{E_p}{E_c}S_t t_p \qquad (6)$$

where f_c is the compressive strength of concrete; E_p and E_c are the elastic modulus of steel plate and concrete, respectively; S_t is the perimeter of loading hammer; t_p is the thickness of steel plate. The details of the calculation of V_{c1}, V_{c2}, V_{s1} and V_{s2} can be found in Huang et al. (2021b).

At point C, the punching cone is fully formed and the top steel plate starts to fail by punching shear fracture. The resistance contribution of the concrete is neglected due to the development of critical diagonal cracks that virtually stops further load transfer into the concrete. The second peak resistance P_2 consists of the resistance contributions of the top steel plate V_{pt}, the middle steel plate V_{pm}, the upper layer shear connectors V_{s1}, and the lower layer shear connectors V_{s2}. i.e.,

$$P_2 = V_{\mathrm{s}} + V_{\mathrm{p}} = V_{\mathrm{s1}} + V_{\mathrm{s2}} + V_{\mathrm{pt}} + V_{\mathrm{pm}} = V_{\mathrm{s1}} + V_{\mathrm{s2}} + S_{\mathrm{t}} t_{\mathrm{p}} \frac{f_{\mathrm{up}}}{\sqrt{3}} + \eta S_{\mathrm{m}} t_{\mathrm{p}} \frac{f_{\mathrm{yp}}}{\sqrt{3}} \tag{7}$$

where η is the degree of composite action; f_{yp} and f_{up} are the yield strength and ultimate strength of steel plate, respectively; S_{m} is the perimeter of the intersection between the punching cone and the middle steel plate.

At point D, the top steel plate is completely punched through and loses its load carrying capacity. The load is directly applied to the lower layer. Thus, the residual load resistance at point D P_{R} is calculated by subtracting V_{pt} and V_{s1} from P_2.

$$P_{\mathrm{R}} = P_2 - V_{\mathrm{pt}} - V_{\mathrm{s1}} = V_{\mathrm{s2}} + \eta S_{\mathrm{m}} t_{\mathrm{p}} \frac{f_{\mathrm{yp}}}{\sqrt{3}} \tag{8}$$

At point E, the middle steel plate starts to fail by punching shear fracture. The third peak load resistance P_3 consists of the contributions of the middle steel plate and the lower layer shear connectors.

$$P_3 = V_{\mathrm{s}} + V_{\mathrm{p}} = V_{\mathrm{s2}} + V_{\mathrm{pm}} = V_{\mathrm{s2}} + S_{\mathrm{m}} t_{\mathrm{p}} \frac{f_{\mathrm{pu}}}{\sqrt{3}} \tag{9}$$

4.2 Stiffness under quasi-static loading

At the elastic stage O-A, the double-layer SCS panel mainly exhibits a global flexural deformation. The global stiffness of the panel, k_{g}, is calculated based on the theory of plates and shells (Timoshenko and Woinowsky-Krieger 1959). However, in order to directly apply the thin plate theory to a thick plate, the stiffness equation needs to be modified by a function that is related to the thickness-to-side length ratio of the panel. In addition, the bond-slip effect between concrete and steel plate also needs to be considered. Thus, the elastic stiffness of the double-layer SCS panel was proposed in reference (Zhang et al. 2021) as follows:

$$k_{\mathrm{e}} = k_{\mathrm{g}} = 0.2\eta\lambda \cdot \frac{D}{0.0116L_{\mathrm{s}}^{2}} = \frac{\eta\lambda D}{0.058L_{\mathrm{s}}^{2}} \tag{10}$$

where λ=-1.65ξ+0.75 and ξ is thickness-to-side length ratio of the panel . D is bending stiffness of the panel and is calculated as the sum of the concrete part D_{c} and the steel part D_{s} by Eqs. (11-13).

$$D=D_{\mathrm{c}}+D_{\mathrm{p}} \tag{11}$$

$$D_{\mathrm{c}}=\frac{E_{\mathrm{c}}}{1-v_{\mathrm{c}}^{2}}\cdot 2\left[\frac{h_{\mathrm{c}}^{3}}{12}+h_{\mathrm{c}}\left(\frac{h_{\mathrm{c}}+t_{\mathrm{p}}}{2}\right)^{2}\right] \tag{12}$$

$$D_{\mathrm{p}}=\frac{E_{\mathrm{p}}}{1-v_{\mathrm{p}}^{2}}\cdot\left[\frac{t_{\mathrm{p}}^{3}}{12}+2\left(\frac{t_{\mathrm{p}}^{3}}{12}+t_{\mathrm{p}}\left(h_{\mathrm{c}}+t_{\mathrm{p}}\right)^{2}\right)\right] \tag{13}$$

where h_{c} is height of a single concrete layer; v_{c} and v_{p} are Poisson's ratio of the concrete and steel plate, respectively.

At the first plastic stage B-C, the total deformation δ_{t} consists of the global bending deformation δ_{g} and the local bulging deformation δ_{b}. The plastic stiffness k_{p1} is calculated by using as a tandem spring model of global bending stiffness k_{g} and local bugling stiffness k_{b}.

$$k_{\mathrm{p1}}=\frac{k_{\mathrm{g}}k_{\mathrm{b}}}{k_{\mathrm{g}}+k_{\mathrm{b}}} \tag{14}$$

where k_{g} is calculated according to Eq. (12), while k_{b} is calculated by Eq. (15) below.

$$k_{\mathrm{b}} = 4\pi\lambda f_{\mathrm{yp}}t_{\mathrm{p}} \tag{15}$$

Based on the test results (Huang et al. 2021b), the slope of the second plastic stage D-E is almost the same as that of the first plastic stage, since both of which are due to the membrane effect of the steel plates. Thus, the second plastic stiffness is assumed the same as the first plastic stiffness, i.e.,

$$k_{\mathrm{p2}}=k_{\mathrm{p1}} \tag{16}$$

Based on the equations of the peak resistances and stiffnesses, the load-displacement curve shown in Figure 12d can be determined. Since a partially composite panel has a longer plateau (A-B) than a fully composite panel, the δ_1' is assumed to be $\delta_1' = \delta_1/\sqrt{\eta'}$, where η' is a stiffness reduction factor and calculated by $\eta' = \sqrt{n_{\mathrm{p}}/n_{\mathrm{f}}}$, where n_{p} and n_{f} are the number of shear connectors of the panel and the number of required shear connectors, respectively (JEAG 2005).

4.3 SDOF model

The SDOF model assumes the drop hammer and SCS panel stick together after contact, as shown in Figure 13a. The initial velocity of the two objects in contact is calculated based on the law of momentum conservation shown in Eq. (17).

$$v_0 = \frac{m_\mathrm{h}}{m_\mathrm{e}} v_{0\mathrm{h}} \tag{17}$$

where $v_{0\mathrm{h}}$ is the impact velocity of the drop hammer; $m_\mathrm{e}=m_\mathrm{s}+m_\mathrm{h}$, in which m_h is the physical mass of the hammer, and m_s is the effective mass of the SCS panel calculated by multiplying the total mass of the panel with a transformation factor, which is dependent on the member geometry, support conditions, and expected response (elastic, elasto-plastic, or plastic). Here this factor is determined as 0.2 by integrating the square of shape function within the area of the panel, and the detailed procedure refers to the literature (Wang et al. 2021a; Bruhl et al. 2015).

The dynamic equation of motion of the SDOF model is then given as:

$$m_\mathrm{e}\ddot{u}_\mathrm{s}(t) + c_\mathrm{s}\dot{u}_\mathrm{s}(t) + k_\mathrm{s}u_\mathrm{s}(t) = 0 \tag{18}$$

where $\ddot{u}_\mathrm{s}(t)$, $\dot{u}_\mathrm{s}(t)$ and $u_\mathrm{s}(t)$ are the acceleration, velocity and displacement of the two objects in contact, respectively. The damping coefficient c_s is ignored during the first loading stage, as the impact velocity at this stage is large while the damping coefficient has marginal influence on the maximum deformation (Bruhl et al. 2015). However, the damping is effective in the following unloading and free vibration stages, as it decreases the amplitude of the deformation rapidly. Thus, the damping coefficient c_s is expressed as:

$$c_\mathrm{s} = \begin{cases} 0 & 0 \le t \le t(u_{\mathrm{s,max}}) \\ \sqrt{m_\mathrm{e}k_{\mathrm{se}}} & t > t(u_{\mathrm{s,max}}) \end{cases} \tag{19}$$

where $t(u_{\mathrm{s,max}})$ is the time when the maximum deformation is reached. The term $k_\mathrm{s}u_\mathrm{s}(t)$ in Eq. (18) can be represented by the load resistance function of the spring s, $P[u_\mathrm{s}(t)]$, as shown in Figure 13b. Thus, Eq. (18) can be modified as:

$$m_\mathrm{e}\ddot{u}_\mathrm{s}(t) + c_\mathrm{s}\dot{u}_\mathrm{s}(t) + P[u_\mathrm{s}(t)] = 0 \tag{20}$$

The solution of the dynamic equation of motion is an initial value problem, which can be solved via the finite difference method. The displacement, velocity and acceleration at step i are calculated, respectively, as:

$$u_{\mathrm{s}}(t_{\mathrm{i}}) = u_{\mathrm{s,i}} \tag{21}$$

$$\dot{u}_{\mathrm{s}}(t_{\mathrm{i}}) = \dot{u}_{\mathrm{s,i}} = \frac{1}{\Delta t}\left(u_{\mathrm{s,i}} - u_{\mathrm{s,i\text{-}1}}\right) \tag{22}$$

$$\ddot{u}_{\mathrm{s}}(t_{\mathrm{i}}) = \ddot{u}_{\mathrm{s,i}} = \frac{1}{\Delta t^2}\left(u_{\mathrm{s,i+1}} - 2u_{\mathrm{s,i}} + u_{\mathrm{s,i\text{-}1}}\right) \tag{23}$$

Substituting Eqs. (21)-(23) to Eq. (20) gives:

$$u_{\mathrm{s,i+1}} = \left[2 - \frac{c_{\mathrm{s}}\Delta t}{m_{\mathrm{e}}}\right]u_{\mathrm{s,i}} - \left[1 - \frac{c_{\mathrm{s}}\Delta t}{m_{\mathrm{e}}}\right]u_{\mathrm{s,i-1}} - \frac{\Delta t^2}{m_{\mathrm{e}}}P(u_{\mathrm{s,i}}) \tag{24}$$

The initial conditions of the dynamic equation of motion are:

$$u_{\mathrm{s,0}}=0 \quad \dot{u}_{\mathrm{s,0}}=v_0 \tag{25}$$

The solution of $u_{\mathrm{s,1}}$ needs $u_{\mathrm{s,0}}$ and $u_{\mathrm{s,-1}}$, which can be determined by considering i=0 in Eq.(22) as,

$$u_{\mathrm{s,\text{-}1}}=u_{\mathrm{s,0}} - \Delta t\dot{u}_{\mathrm{s,0}} = -\Delta t v_0 \tag{26}$$

4.4 TDOF model

The TDOF model considers the drop hammer and the SCS panel as two independent masses, and the contact between the two is represented by an additional set of spring and damper, as shown in Figure 14a. The resistance functions of the spring s and spring h in the TDOF model are shown in Figure 14b and 14c, where the function of the spring s is the same as that in the SDOF model. The deformation of the spring h is the relative displacement between the drop hammer and the SCS panel, i.e., u_{h}-u_{s}. Wang et al. (2021a) derives the expression of k_{h} at the elastic stage as below:

$$k_{\mathrm{he}} = \frac{k_{\mathrm{b}}k_{\mathrm{i}}}{k_{\mathrm{b}} - k_{\mathrm{i}}} \tag{27}$$

where k_{b} is the local bulging stiffness and is calculated according to Eq. (15). k_{i} is the local indentation stiffness, and is calculated by Eq. (28).

$$k_{\mathrm{i}} = 2.5\pi f_{\mathrm{yp}}t_{\mathrm{p}} + 2\pi r f_{\mathrm{c}} \tag{28}$$

where r is the radius of the hammer.

The plastic stiffness k_{hp} and unloading stiffness k_{hu} are simplified as the elastic stiffness multiplied by two factors, α and β, respectively. The values of α and β are 0.4 and 5.0, respectively (Wang et al. 2021a). The reason for using a larger unloading stiffness is that the local indentation is hardly to recover during unloading.

The dynamic equation of motion of the TDOF model is then given as:

$$\begin{cases} m_{\mathrm{s}}\ddot{u}_{\mathrm{s}}(t)+c_{\mathrm{s}}\dot{u}_{\mathrm{s}}(t)+P_{\mathrm{s}}[u_{\mathrm{s}}(t)]=c_{\mathrm{h}}[\dot{u}_{\mathrm{h}}(t)-\dot{u}_{\mathrm{s}}(t)]+P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)] \\ m_{\mathrm{h}}\ddot{u}_{\mathrm{h}}(t)+c_{\mathrm{h}}[\dot{u}_{\mathrm{h}}(t)-\dot{u}_{\mathrm{s}}(t)]+P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)]=0 \end{cases} \tag{29}$$

where m_{h} is the physical mass of the hammer, and m_{s} is the effective mass of the SCS panel and is 0.2 times of the physical mass, similar to that used in the SDOF model.

The TDOF model is capable to calculate the contact force between the drop hammer and the SCS panel.

$$F(t)=m_{\mathrm{h}}\ddot{u}_{\mathrm{h}}(t)=-c_{\mathrm{h}}[\dot{u}_{\mathrm{h}}(t)-\dot{u}_{\mathrm{s}}(t)]-P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)] \tag{30}$$

Using the finite difference method, the solution of the dynamic equation of motion is derived as below:

$$\begin{cases} u_{\mathrm{s,i+1}}=2u_{\mathrm{s,i}}-u_{\mathrm{s,i-1}}+\dfrac{\Delta t}{m_{\mathrm{s}}}\Big[c_{\mathrm{h}}\left(u_{\mathrm{h,i}}-u_{\mathrm{h,i-1}}-u_{\mathrm{s,i}}+u_{\mathrm{s,i-1}}\right)-c_{\mathrm{s}}\left(u_{\mathrm{s,i}}-u_{\mathrm{s,i-1}}\right)\Big]+\dfrac{\Delta t^2}{m_{\mathrm{s}}}\Big[P_{\mathrm{h}}(u_{\mathrm{h,i}}-u_{\mathrm{s,i}})-P_{\mathrm{s}}(u_{\mathrm{s,i}})\Big] \\ u_{\mathrm{h,i+1}}=2u_{\mathrm{h,i}}-u_{\mathrm{h,i-1}}-\dfrac{\Delta t}{m_{\mathrm{h}}}c_{\mathrm{h}}\left(u_{\mathrm{h,i}}-u_{\mathrm{h,i-1}}-u_{\mathrm{s,i}}+u_{\mathrm{s,i-1}}\right)-\dfrac{\Delta t^2}{m_{\mathrm{h}}}P_{\mathrm{h}}(u_{\mathrm{h,i}}-u_{\mathrm{s,i}}) \\ F_{\mathrm{i+1}}=-\dfrac{c_{\mathrm{h}}}{\Delta t}\left(u_{\mathrm{h,i+1}}-u_{\mathrm{h,i}}-u_{\mathrm{s,i+1}}+u_{\mathrm{s,i}}\right)-P_{\mathrm{h}}(u_{\mathrm{h,i+1}}-u_{\mathrm{s,i+1}}) \end{cases} \tag{31}$$

where the resistance functions, $P_{\mathrm{h}}(u_{\mathrm{h,i}}\text{-}u_{\mathrm{s,i}})$ and $P_{\mathrm{s}}(u_{\mathrm{s,i}})$, follow the indications in Figure 14b and 14c, respectively. The damping coefficients c_{s} and c_{h} are ignored during the first loading stage and are only effective in the subsequent unloading stages. In addition, c_{h} is also ignored after the hammer detached from the top steel plate. The expressions of c_{s} and c_{h} are determined as:

$$c_{\mathrm{s}}=\begin{cases} 0 & 0\le t\le t(u_{\mathrm{s,max}}) \\ \sqrt{m_{\mathrm{s}}k_{\mathrm{se}}} & t>t(u_{\mathrm{s,max}}) \end{cases} \tag{32}$$

$$c_{\mathrm{h}}=\begin{cases} 0 & 0\le t\le t(u_{\mathrm{h,max}})\,\&\,P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)]>0 \\ \sqrt{\dfrac{m_{\mathrm{h}}m_{\mathrm{s}}}{m_{\mathrm{h}}+m_{\mathrm{s}}}k_{\mathrm{he}}} & t>t(u_{\mathrm{h,max}})\&P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)]>0 \\ 0 & t>t(u_{\mathrm{h,max}})\&P_{\mathrm{h}}[u_{\mathrm{h}}(t)-u_{\mathrm{s}}(t)]\le 0 \end{cases} \tag{33}$$

The TDOF model can be solved by imposing the following initial conditions.

$$\begin{cases} u_{s,0}=0 & \dot{u}_{s,0}=0 & u_{s,-1}=u_{s,0}-\Delta t\dot{u}_{s,0}=0 \\ u_{h,0}=0 & \dot{u}_{h,0}=v_{0h} & u_{s,-1}=u_{s,0}-\Delta t\dot{u}_{s,0}=-\Delta t v_{0h} \end{cases} \tag{34}$$

4.5 Verification

Table 6 lists the key parameters of the SDOF model and the TDOF model calculated from the above equations. Figure 15 compares the displacement-time and load-time responses, respectively, between the test results, the FE results and the results predicted by the analytical models. Table 7 compares the main characteristics of the response curves, including the peak deformation δ_p, residual deformation δ_r, and peak impact force F_p, obtained from the FE model, the SDOF model and the TDOF model. The SDOF model is only capable of predicting displacement but unable to predict the contact force since the hammer and panel are considered to be in constant contact. Compared to the TDOF model, the SDOF model may overpredict the peak deformation of the double-layer SCS panels. This may be attributed to the energy conversion mechanism during the impact. In this study, the ratio of the hammer weight to the effective panel weight (m_h/m_s) is about 15, thus the hammer is a much heavier object compared to the effective panel. After the hammer impacts the panel, the energy lost during the impact is very small based on the law of momentum conservation. Thus, the initial kinetic energy in the SDOF model is almost the same as that in the TDOF model. As the damping is neglected during the first loading stage for both the SDOF model and TDOF model, the initial kinetic energy of the mass is completely converted to the potential energy of the spring. However, part of the energy is distributed to spring k_h in the TDOF model, while the energy distributed to spring k_s in the SDOF model is larger than that in the TDOF model. As a result, k_s of the spring predicted by the SDOF model is larger than that predicted by the TDOF model.

The comparison shows that both the FE model and TDOF model have a very good prediction on the impact response of the double-layer SCS panels as shown by the displacement-time and load-time curves. The mean values and standard deviations of the ratios to test results are within an acceptable range. In addition, the results predicted by the FE model and TDOF model follow a more reasonable trend on the influence of the volume proportion of rubber power, which is not clearly shown by the test results as discussed in Section 2.9. Both the numerical and analytical results have firmly confirmed that an increase of the volume proportion of rubber powder would slightly increase the peak deformation of the panel, but slightly decrease the peak impact force, as reflected in Table 7 for the peak deformation and peak impact force of SULCS-150-6(DH), SR5ULCS-150-6(DH), and SR10ULCS-150-6(DH).

5. Conclusion

This study has examined the impact response of double-layer Steel-RULCC-Steel sandwich panels through experimental, numerical and analytical approaches. Seven full-scale sandwich panels with varying material and geometric parameters have been tested under drop hammer impact. Advanced FE simulation has been performed in LS-DYNA to extract more information on the strain, stress and energy consumption inside the panel during the impact. Analytical SDOF model and TDOF model have been developed to predict the impact responses for double-layer SCS panels based on the proposed quasi-static load-displacement relationship. The following conclusions are obtained:

(1) Compared to the single-layer panel, the double-layer panel provides better protection against impact. The middle steel plate spreads the loading to a larger area inside the panel, thus more materials are motivated to absorb the impact energy. Although the differences of peak displacement and load between the single-layer panel and double-layer panel are small, the damage at failure of the single-layer panel is severer than that of the double-layer panel.

(2) The SCS panel infilled with RULCC exhibits better impact behavior than the panel infilled with ULCC, as the addition of rubber powder increases the energy absorption ability of the concrete. The recommended replacing volume proportion of rubber powder is 5% of the fine aggregate. The partially composite panel has a better deformation ability than the fully composite panel, as the SCS panel with smaller degree of composite action is more flexible and ductile due to the larger bond-slip between the steel plate and concrete.

(3) The steel plate and the concrete consume more than 95% of the impact energy, in the form of steel yielding and concrete cracking. The decrease of degree of composite action increases the energy consumption ratio of steel plate

but decreases the energy consumption ratio of concrete, as a greater number of cracks are produced in the fully composite panel than that in the partially composite panel.

(4) The SDOF model is unable to predict the contact force between the drop hammer and SCS panel and overpredicts the peak displacement at the bottom center of the panel if the hammer weight is much larger than the weight of the effective panel. In contrast, the developed TDOF model provides a much more accurate prediction on the impact response of the double-layer SCS panels, including the peak deformation, residual deformation, and peak impact force.

(5) The current study mainly focuses on the double-layer SCS panel subjected to a single impact loading. In real life service of a structures, it may be subjected to multiple impacts from, e.g., vehicles or other subjects. In this case, an accurate assessment of multiple impact performance and residual resistance, etc., is essential to facilitate fast-track repairing or retrofitting. Therefore, future research on the respect is urgently required.

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Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

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Table 1: Geometric dimensions of test specimens

Specimen	Layer number	h_c (mm)	t_p (mm)	h_t (mm)	s (mm)	L (mm)	L_s (mm)	J-hook (mm)	Headed stud (mm)	ρ (%)	η
SULCS-150-9(SH)	1	141	9.2	159.4	150	1200	1000	Ø12@300	Ø13@300	11.5	0.43
SULCS-100-6(DH)	2	70	5.8	157.4	100	1200	1000	Ø12@200	Ø13@200	11.1	1.00
SULCS-150-6(DH)	2	70	5.8	157.4	150	1200	1000	Ø12@300	Ø13@300	11.1	0.64
SULCS-200-6(DH)	2	70	5.8	157.4	200	1200	1000	Ø12@400	Ø13@400	11.1	0.37
SULCS-150-6(DJ)	2	70	5.8	157.4	150	1200	1000	Ø12@150	/	11.1	0.64
SR5ULCS-150-6(DH)	2	70	5.8	157.4	150	1200	1000	Ø12@300	Ø13@300	11.1	0.64
SR10ULCS-150-6(DH)	2	70	5.8	157.4	150	1200	1000	Ø12@300	Ø13@300	11.1	0.64

Notes: h_c=height of one-layer concrete; t_p=thickness of steel plate; h_t=total height of SCS panel; s=spacing between shear connectors; L=edge length of SCS panel; L_s=length between two support lines; J-hook Ø12@300 indicates the J-hook diameter is 12mm and the spacing between two J-hooks is 300mm; Headed stud Ø13@300 indicates the headed stud diameter is 13mm and the spacing between two headed studs is 300mm; $\rho=2t_p/h_t$ for single-layer specimen, and $3t_p/h_t$ for double-layer specimen, indicating the steel contribution ratio; η is the degree of composite action.

Table 2: Mix proportion of ULCC and RULCC (kg/m^3)

	W	OPC	SF	GGBFS	R	F	HWRA	SRA
ULCC	259.0	702.0	78.0	339.9	/	5.8	12.0	9.0
R5ULCC	259.0	702.0	78.0	322.9	18.8	5.8	12.4	9.0
R10ULCC	259.0	702.0	78.0	305.9	37.7	5.8	12.8	9.0

Notes: W=water; OPC=ordinary Portland cement; SF=silica fume; GGBFS=mineral powder; R=rubber powder; F=steel fiber; HWRA=high Water reducing agent; SRA=shrinkage reducing agent.

Table 3: Material properties of ULCC and RULCC

Mix	Density (kg/m^3)	Cube compressive strength (MPa)	Cylinder compressive strength (MPa)	Elastic modulus (GPa)	Poisson's ratio	Tensile strength (MPa)	Maximum tensile strain (με)
ULCC	1388	49.2	35.6	11.8	0.28	2.9	0.0029
R5ULCC	1296	44.3	33.2	9.9	0.30	3.1	0.0031
R10ULCC	1203	40.2	31.0	8.5	0.33	3.2	0.0039

Table 4: Material properties of steel

Steel component	Yield strength (MPa)	Yield strain (με)	Ultimate strength (MPa)	Ultimate strain (με)	Elastic modulus (GPa)	Poisson's ratio
Steel plate (t_s=5.8mm)	288	2034	440	0.27	202	0.30
Steel plate (t_s=9.2mm)	292	1994	460	0.31	208	0.30
J-hook	463	2034	690	0.15	204	0.30
Headed stud	772	4972	916	0.11	205	0.30

Table 5: Impact parameters and test results

Specimen	m_h (kg)	H (m)	v_{0h} (m/s)	P (kg·m/s)	E (J)	F_p (kN)	δ_p (mm)	δ_r (mm)	Failure mode
SULCS-150-9(SH)	1012	5	9.8	9948.0	48894.2	941.9	43.6	33.8	Fracture
SULCS-100-6(DH)	1012	5	9.8	9920.6	48626.0	1063.4	32.8	25.4	Indentation
SULCS-150-6(DH)	1012	5	9.9	10001.6	49422.9	960.2	43.3	32.4	Indentation
SULCS-200-6(DH)	1012	5	9.9	9998.6	49392.9	907.7	47.4	35.1	Indentation
SULCS-150-6(DJ)	1012	5	9.9	9978.3	49193.1	874.3	45.5	34.3	Indentation
SR5ULCS-150-6(DH)	1012	5	9.9	10008.7	49492.9	884.4	46.5	34.5	Indentation
SR10ULCS-150-6(DH)	1012	5	9.9	10008.7	4942.9	938.3	41.1	30.4	Indentation

Notes: m_h=hammer mass; H=drop height of the hammer; v_{0h}, P, E=impact velocity, impact momentum and impact energy of the hammer right before contacting the panel; F_p=peak load on the panel; δ_p=peak deformation at the bottom center of the panel; δ_r=residual deformation at the bottom center of the panel.

Table 6: Key parameters in the analytical models

Specimen	m_s	m_h	P_1	P_2	P_R	P_3	SDOF			TDOF		
							k_{se}	k_{sp}	c_s	k_{he}	c_s	c_h
SULCS-100-6(DH)	70.0	1012	648.9	1484.1	813.4	1060.4	158.0	12.5	413.5	51.4	105.2	58.0
SULCS-150-6(DH)	68.8	1012	620.2	1200.1	798.6	953.8	102.6	11.9	333.0	51.4	84.0	57.5
SULCS-200-6(DH)	68.0	1012	600.6	983.2	787.1	913.7	58.4	11.1	251.1	51.4	63.0	57.2
SULCS-150-6(DJ)	68.8	1012	617.6	1189.5	793.3	948.9	102.6	11.9	333.0	51.4	84.0	57.5
SR5ULCS-150-6(DH)	66.2	1012	611.4	1191.6	794.4	951.8	99.9	11.9	328.2	51.0	81.3	56.3
SR10ULCS-150-6(DH)	63.6	1012	605.5	1184.5	790.8	950.8	97.9	11.8	324.5	50.7	78.9	55.1

Notes: The unit of mass is kg, the unit of load is kN, the unit of stiffness is kN/mm, and the unit of damping is kN·s/m.

Table 7: Verification of the FE model, SDOF model and TDOF model

Specimen	Test	FE	FE/Test	SDOF	SDOF/Test	TDOF	TDOF/Test
	δ_p (mm)						
SULCS-100-6(DH)	32.8	34.7	1.06	45.4	1.38	34.8	1.06
SULCS-150-6(DH)	43.3	44.4	1.03	54.1	1.25	42.9	0.99
SULCS-200-6(DH)	47.4	50.4	1.06	57.7	1.22	48.0	1.01
SULCS-150-6(DJ)	45.5	44.9	0.99	53.3	1.17	44.6	0.98
SR5ULCS-150-6(DH)	46.5	45.0	0.97	54.1	1.16	45.3	0.97
SR10ULCS-150-6(DH)	41.1	45.3	1.10	54.7	1.33	45.7	1.11
Mean			1.03		1.25		1.02
Std.dev			0.05		0.09		0.05
	δ_r (mm)						
SULCS-100-6(DH)	25.4	26.7	1.05	36.9	1.45	28.3	1.11
SULCS-150-6(DH)	32.4	34.3	1.06	42.9	1.32	33.3	1.03
SULCS-200-6(DH)	35.1	38.6	1.10	43.5	1.24	34.2	0.97
SULCS-150-6(DJ)	34.3	34.9	1.02	41.7	1.22	35.2	1.03
SR5ULCS-150-6(DH)	34.5	34.2	0.99	42.3	1.23	35.4	1.03
SR10ULCS-150-6(DH)	30.4	34.5	1.13	42.5	1.40	36.1	1.19
Mean			1.06		1.31		1.06
Std.dev			0.05		0.10		0.08
	F_p (kN)						
SULCS-100-6(DH)	1063.4	1075.7	1.01	/	/	1065.6	1.00
SULCS-150-6(DH)	960.2	995.2	1.04	/	/	973.4	1.01
SULCS-200-6(DH)	907.7	911.1	1.00	/	/	932.5	1.03
SULCS-150-6(DJ)	874.3	950.8	1.09	/	/	957.8	1.10
SR5ULCS-150-6(DH)	884.4	988.5	1.12	/	/	948.1	1.07
SR10ULCS-150-6(DH)	938.3	984.6	1.05	/	/	943.0	1.01
Mean			1.05				1.04
Std.dev			0.04				0.04